RETROSPECTIVE EVALUATION OF EARTHQUAKE-DAMAGED REINFORCED-CONCRETE BUILDINGS USING PRACTICAL METHODS

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Abstract

Robust seismic performance evaluation methods are important for quantifying the vulnerability of existing infrastructure and designing appropriate rehabilitation strategies. In United States practice, ASCE 41 (*Seismic Evaluation and Retrofit of Existing Buildings*) guides the evaluation process for buildings, and portions of this document have been adopted or referenced internationally. Provisions for modeling recommendations and performance acceptance criteria are at the core of ASCE 41; few other codified resources are as complete. For reinforced-concrete buildings in particular, ASCE 41 specifies expected material properties, effective stiffness values, backbone curves, and acceptance criteria for columns, beams, beam-column joints, and shear walls that are well founded in previous experimental research at the component or subassemblage level. However, advancements in these modeling recommendations have not been developed or adopted consistently across all components, and their robustness in the context of a full building system has not been evaluated extensively.

To improve the understanding of the quality of the ASCE 41 modeling recommendations and acceptance criteria, the National Institute of Standards and Technology (NIST) and the Applied Technology Council (ATC) initiated a project entitled "Performance-Based Seismic Engineering: Benchmarking of Existing Building Evaluation Methodologies" (ATC-134). In this project, the latest evaluation standard in the United States, ASCE 41-17, is employed to model reinforced-concrete buildings damaged in earthquakes (real and experimentally simulated). The results of both linear and nonlinear dynamic procedures are compared against observed damage.

This paper examines two buildings evaluated in this study that were designed and built in the 1960s. The first building, the Nanhua District Office in Tainan, Taiwan, had three stories with moment-resisting frames and masonry infill (full and partial height). This building sustained severe column damage in the 2016 Meinong Earthquake, leading to its demolition. The second building, the Pyne Gould Corporation Building in Christchurch, New Zealand, had five stories with shear walls at the core. This building collapsed in the 2011 Christchurch Earthquake due to failure of the walls and gravity system. The buildings were modeled using commercial software (linear procedure) and *OpenSees* (nonlinear procedure) and analyzed using appropriate ground motions from their respective events. In *OpenSees*, the models primarily employed fiber-based beam-column elements. The columns were modeled with a "limit state" spring in series with the fiber element that was newly developed to bound the response based on a dynamically updated ASCE 41-17 backbone curve. The results show limitations of the ASCE 41-17 modeling recommendations, including column effective stiffness values and backbone curve parameters; these limitations may inhibit accurate prediction of the global response mechanism and potentially misguide retrofit design. Damage-prediction accuracy for each studied building, recommendations for future evaluation provisions, and modeling considerations in practice will also be discussed.

Keywords: Reinforced concrete; evaluation; existing infrastructure; case study; nonlinear modeling.

1. Introduction

Reinforced-concrete (RC) buildings are pervasive in existing infrastructure around the world, and the older RC building stock is the subject of increasing scrutiny from the engineering community as public awareness of seismic vulnerability grows. Evidence from past earthquakes and an extensive body of previous research has shown that many vintage RC buildings are potentially nonductile. Under lateral loading induced by moderate and large earthquakes, nonductile RC buildings may sustain collapse or significant damage that delays recovery. This vulnerability is potentially addressed through seismic retrofit, and widespread engineering concern over the performance of the existing infrastructure has prompted notable large-scale retrofit programs in Taiwanese schools following the 1999 Chi-Chi Earthquake [1] and Los Angeles [2], where a mandatory retrofit ordinance for nonductile RC buildings has been enacted.

The motivation for seismic retrofit of RC buildings is clear and well founded, but determining the retrofit need and strategy for an individual building is technically challenging. The American Society of Civil Engineers (ASCE) standard entitled *Seismic Evaluation and Retrofit of Existing Buildings* (ASCE/SEI 41-17) [3], provides codified frameworks to perform these tasks across different levels of complexity. At the simplest level of evaluation in ASCE/SEI 41-17, Tier 1, buildings can be screened to identify the extent of evaluation required relatively quickly. This procedure is useful for identifying buildings that clearly do *not* require retrofit, but it is noted that buildings that fail Tier 1 screening do not necessarily require retrofit; hence, a more advanced evaluation procedure is required. The most rigorous level of evaluation in ASCE/SEI 41-17 is defined as "Tier 3," which encompasses linear or nonlinear system analysis procedures using either static or dynamic loading to simulate seismic load effects. Since a Tier 3 evaluation requires construction of a numerical model of the building, ASCE/SEI 41-17 also provides comprehensive modeling procedures for buildings. This modeling guidance has gained popularity beyond US practice and evaluation of existing buildings.

Despite the requirement of system-level modeling, the ASCE/SEI 41-17 Tier 3 modeling procedures and acceptance criteria are largely defined and evaluated at the component level. For RC buildings, these components include columns, beams, beam-column joints, and shear walls. The procedures include provisions for material properties and effective stiffness values to define fundamental component properties (strength and stiffness), but the document is centered on prescribed backbone curves for nonlinear component response, such as that shown in Fig. 1. The backbone-curve concept generally guides performance acceptance criteria as well. Acceptance criteria are defined for immediate occupancy (IO), life safety (LS), and collapse prevention (CP) damage states for each component; for RC columns, the relationship between the acceptance criteria and general backbone curve is shown in Fig. 1. In linear procedures, these acceptance criteria are "*m*-factors" that use the equal-displacement concept to relate computed elastic forces or moments to inelastic deformations. It is noted that component limit states that do not have prescribed *m*-factors or deformation limits have binary, force-based acceptance criteria (i.e., there is no assumed ductility).

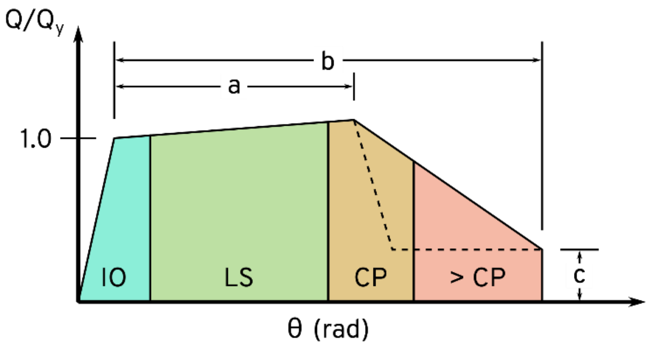


Fig. 1 – Typical ASCE/SEI 41-17 backbone curve with acceptance criteria

These component modeling parameters and their corresponding acceptance criteria were calibrated from experimental data collected primarily from component and subassemblage testing, but their applicability in the context of a building system in ASCE/SEI 41-17 is not clear due to several key issues:

- Building components in earthquakes are subjected to demands that are not often considered in component and subassemblage testing (e.g., variable axial load or biaxial loading of columns);
- Experimental specimens are generally tested under ideal conditions with well-controlled construction processes, well-defined boundary conditions, and highly regular but severe loading; and
- Modeling procedures differ across components with respect to implementation method (e.g., choice of deformation parameter) and rate of code advancements.

Therefore, there is a need to validate the system modeling approach in ASCE/SEI 41-17 to evaluate its strengths and weaknesses and guide its development, especially as its use is expected to increase to address the vulnerability of the existing infrastructure. The Applied Technology Council (ATC) project entitled "Performance-Based Seismic Engineering: Benchmarking of Existing Building Evaluation Methodologies" (ATC-134) seeks to, in part, evaluate the ASCE/SEI 41-17 Tier 3 analysis procedures using real earthquake-damaged RC buildings as case studies. This paper discusses existing and new methods for modeling various RC components in ASCE/SEI 41-17 that are applied to evaluate the predicted response of the Nanhua District Office (damaged in the 2016 Meinong Earthquake in Taiwan) and the Pyne Gould Corporation Building (collapsed in the 2011 Christchurch Earthquake in New Zealand) under the aforementioned ATC-134 project. The Nanhua District Office was evaluated with linear and nonlinear response-history analysis procedures, whereas the Pyne Gould Corporation Building was evaluated with only the nonlinear response-history analysis procedure. These two building investigations identify the challenges of using the ASCE/SEI 41-17 modeling procedures, sources of inaccuracies in predicted response, and opportunities for improving the standard.

2. Case Study: Nanhua District Office

2.1 Background

The Nanhua District Office was a three-story moment-resisting frame with masonry infill walls that was severely damaged in the 2016 Meinong Earthquake, a M_w6.4 earthquake which induced strong ground shaking across Southern Taiwan, most notably in the Tainan region [4]. The acceleration response spectra for this earthquake are shown in Fig. 2 for the recording at Station A730, located approximately 2.5 km from the building site. This recording best estimates the ground shaking at the building site; in Fig. 2, the horizontal acceleration record pairs are oriented to match the building's principal axes.

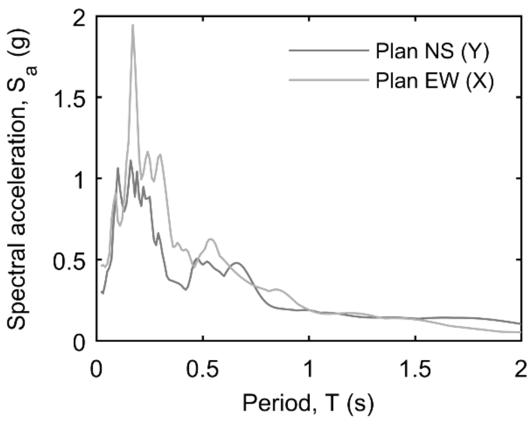
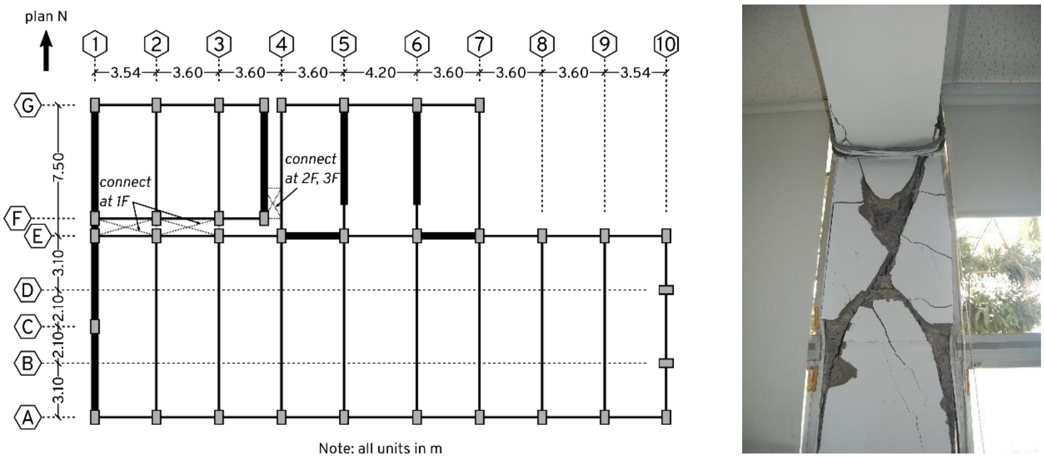


Fig. 2 –2016 Meinong Earthquake acceleration response spectra from Station A730 recording

Like many of the buildings damaged in this event, the Nanhua District Office was an older (late 1960s) RC building that sustained a nonductile, soft-story response. The plan sketch of the building in Fig. 3a shows that the building was fairly regular in plan, but the columns had limited transverse reinforcement and were effectively shortened by partial-height masonry infill on the exterior frames of the building. The building's primary damage was to the first story, where large, residual diagonal cracks, as in Fig. 3b, were reported in the east-west (EW) direction of most columns. This crack pattern is indicative of a shear or flexure-shear failure mode. There was no apparent residual drift or axial-load failure. Several full-height masonry infill walls also had diagonal cracking; these were nonstructural elements, but they contributed to the lateral strength and stiffness of the building. Due to the severe column damage, the building was demolished and replaced.



(a) Plan sketch (thick lines indicate full-height masonry infill wall locations)

(b) Typical first-story column damage (Column A2 shown)

Fig. 3 – Building geometry and characteristic column damage

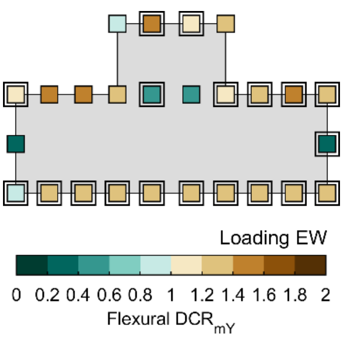
2.2 Linear Response-History Analysis

Linear response-history analysis was performed for the Nanhua District Office using a three-dimensional model of the building developed in *SAP2000* [5]. Beams and columns were modeled with frame elements with the reduced (i.e., effective or cracked) stiffness values required by ASCE/SEI 41-17. Partial-height and full-height masonry infill was modeled using compression-only diagonal struts across the infill in a given bay. It is noted that ASCE/SEI 41-17 does not provide specific guidance for modeling of partial-height infill. The diaphragm was modeled using shell elements to represent RC slabs at each floor. The building was analyzed using the ground motion recorded from Station A730 (Fig. 2). Sumerall [6] provides additional information on the modeling approach and an investigation of alternative approaches for simulating the infill walls.

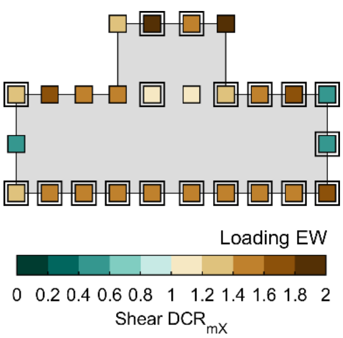
Fig. 4 shows heatmaps of column demand-to-capacity ratios (DCRs) divided by the corresponding *m*-factor for the collapse-prevention damage state (termed DCR_m) for flexural and shear actions in the EW direction on the first story. Hence, a DCR_m value exceeding unity suggests the column exceeds the collapse-prevention damage state. Boxed columns in the figure indicate locations of observed damage from post-earthquake reconnaissance. The plots show a reasonable distribution of predicted damage relative to the observed damage. The magnitude of the flexure and shear DCR_m values are similar with many exceeding unity; this suggests (1) a serious vulnerability to collapse even though the actual building did not collapse and (2) ambiguity in the predicted component failure mode. Consequently, the linear response-history analysis procedure over-predicts the column damage and does not provide guidance for retrofit beyond identifying vulnerable components for this building. More sophisticated analysis (i.e., nonlinear response-history analysis) would be required to enhance confidence in the predicted column failure mode and determine an appropriate retrofit strategy.

2.3 Nonlinear Response-History Analysis

Companion three-dimensional models of the Nanhua District Office were developed in *OpenSees* [7] to evaluate the accuracy of the ASCE/SEI 41-17 nonlinear response-history analysis procedure. The beams and columns were modeled with concentrated hinges at the member ends with either moment-rotation springs or a fiber-limit state hinge. In the moment-rotation spring approach, nonlinear springs with the general backbone curve shown in Fig. 1 are utilized. The strength and deformation parameters of the backbone curves are based on the maximum dynamic axial load and are determined through iterative response-history analysis to converge on the parameters. In the fiber-limit state approach, a zero-length fiber section is used in series with a limit-state spring, a concept extended here from initial work by Elwood [8]. The limit-state spring tracks the total rotation of the hinge region and the maximum axial load demand to dynamically update the prescribed ASCE/SEI 41-17 backbone curve and trigger strength degradation. Prior to strength degradation, the nonlinear response of the hinge region is controlled by the fiber section. This approach eliminates the need to perform iterative response-history analyses and inherently simulates axial-flexural interaction. As in the previous *SAP2000* [5] model, the masonry infill was modeled using compression-only diagonal struts. These struts are represented as dotted lines in Fig. 5, which also shows the first mode shape of the building. Note that this first mode shape is consistent with the observed damage location (first story) and direction (EW). The diaphragm was modeled using a rigid diaphragm constraint. These buildings were subjected to the same rotated A730 ground-motion pair used in the linear response-history analysis.



(a) Flexure



(b) Shear

Fig. 4 – Nanhua District Office linear-procedure demand-to-capacity ratios at first story

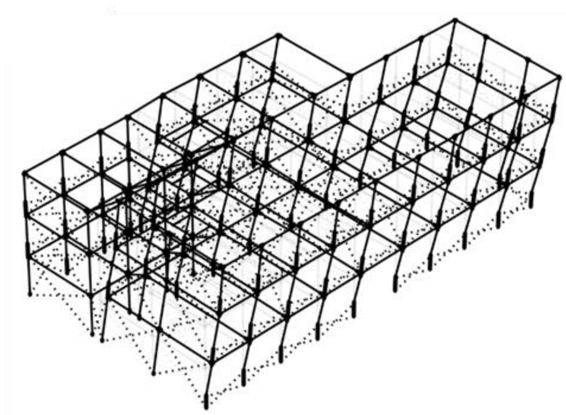
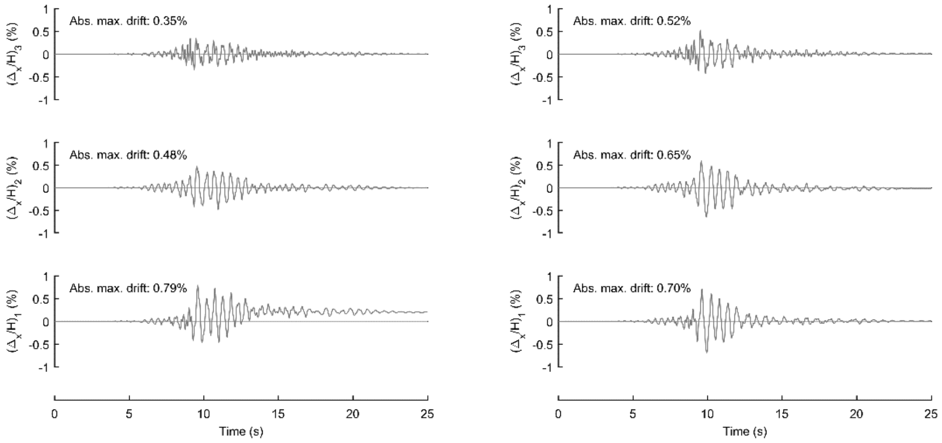


Fig. 5 – Nanhua District Office *OpenSees* model (first mode shape shown)

Figs. 6a and 6b show the respective story-drift response histories for the models using moment-rotation springs and fiber-limit state hinges. While the maximum story drifts are low (less than 1%) and largest on the first story in both cases, the moment-rotation spring model concentrates more damage on the first story. In the fiber-limit state model, the damage is more evenly distributed along the height. The former response is most consistent with the observed damage, as virtually no column damage was reported on the upper stories. This difference in response is due in part to the moment-rotation responses of the springs versus the fiber sections, since the fiber-section stiffness deteriorates significantly between the yield rotation and the rotation at the nominal flexural strength. In contrast, the moment-rotation spring has constant stiffness up to nominal flexural strength. This effect is pronounced on the upper stories where the axial load in the column is relatively low.



(a) Moment-rotation spring model (b) Fiber-limit state model

Fig. 6 – Nanhua District Office story-drift response histories

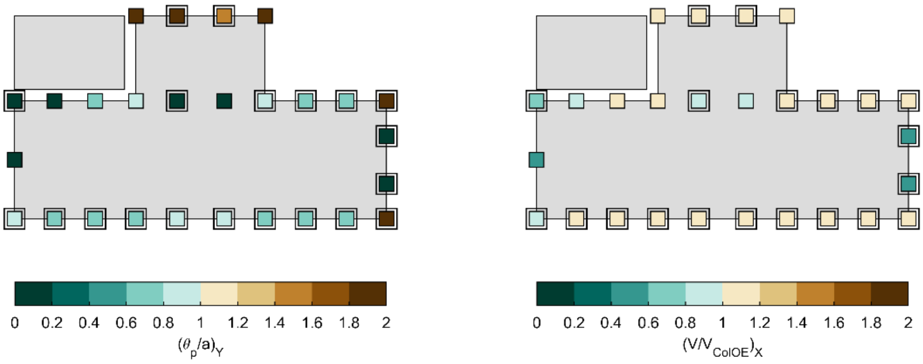
Heatmaps of column DCRs for flexure (left) and shear (right) are shown in Fig. 7. Note that the flexural DCR provided is the ratio of the plastic rotation demand to the prescribed "a" value for the column, which corresponds to the onset of degradation (see Fig. 1). Recall that in the moment-rotation spring model, "a" is defined using the maximum axial load, whereas in the fiber-limit state model, "a" is updated continuously using the maximum historic axial load. As expected based on the story-drift response histories in Fig. 6, the columns in the moment-rotation spring model sustain more damage. However, in both models, the extent of predicted column damage is significantly less than observed. This may suggest that the ASCE/SEI 41-17 backbone curve overpredicts the deformation capacity of the given columns or that the ground motion at Station A730 does not characterize the ground motion at the building site well.

3. Case Study: Pyne Gould Corporation Building

3.1 Background

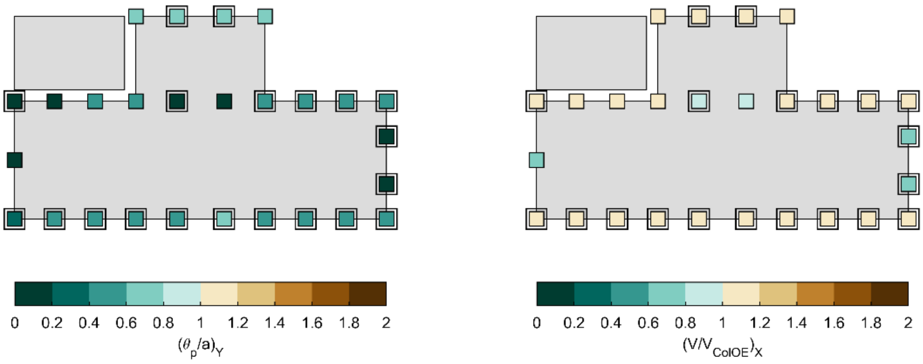
The Pyne Gould Corporation Building was a five-story RC building shear walls at the core that collapsed in the 2011 Christchurch Earthquake in New Zealand. This M_w6.2 earthquake caused significant damage across the city of Christchurch; it was preceded by the larger-magnitude but more-distant 2010 Darfield Earthquake as part of the 2010-2011 Canterbury Earthquake Sequence. The best-estimate recording of the ground motion at the building site is from the Resthaven (REHS) station, for which the acceleration response spectra are shown in Fig. 8.

The Pyne Gould Corporation Building was of similar vintage to the Nanhua District Office (1960s) and had small, lightly reinforced columns and shear walls. Moreover, the building had significant vertical discontinuity at the first-floor interface (between the first and second stories). As shown in Fig. 9, the upper-story columns overhang the first-story columns on the exterior of the building. In addition, the wall area on the first story is significantly greater. Collapse was most likely precipitated by plastic hinging and axial failure of the wall at the base of the second story [9]. As this occurred, the wall acted as a rigid "strongback" above the second story, enforcing approximately equal, large deformation demands along the height. This resulted in extensive axial failures in the beam-column joint regions and fracture of the slab-wall reinforcement.



(a) Flexure, moment-rotation spring model

(b) Shear, moment-rotation spring model



(c) Flexure, fiber-limit state model

(d) Shear, fiber-limit state model

Fig. 7 – Nanhua District Office nonlinear-procedure demand-to-capacity ratios at first story

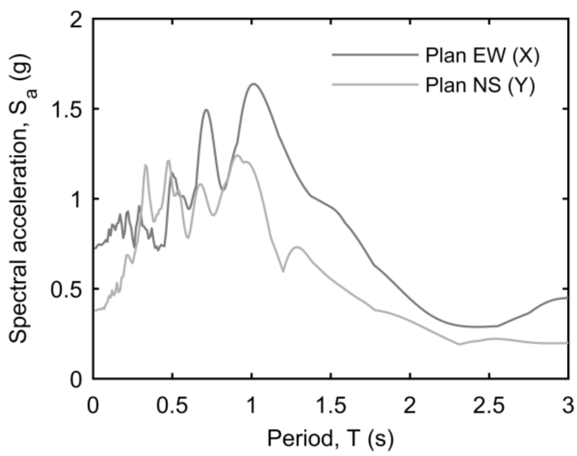


Fig. 8 –2011 Christchurch Earthquake acceleration response spectra from Station REHS recording

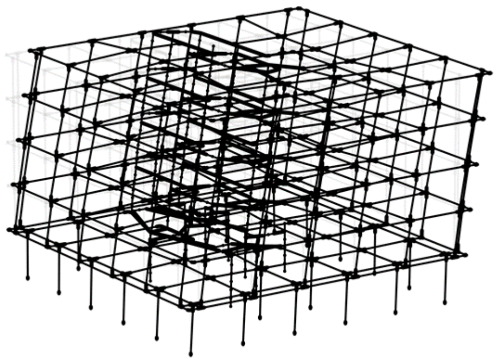


Fig. 9 – Pyne Gould Corporation Building *OpenSees* model (first mode shape shown)

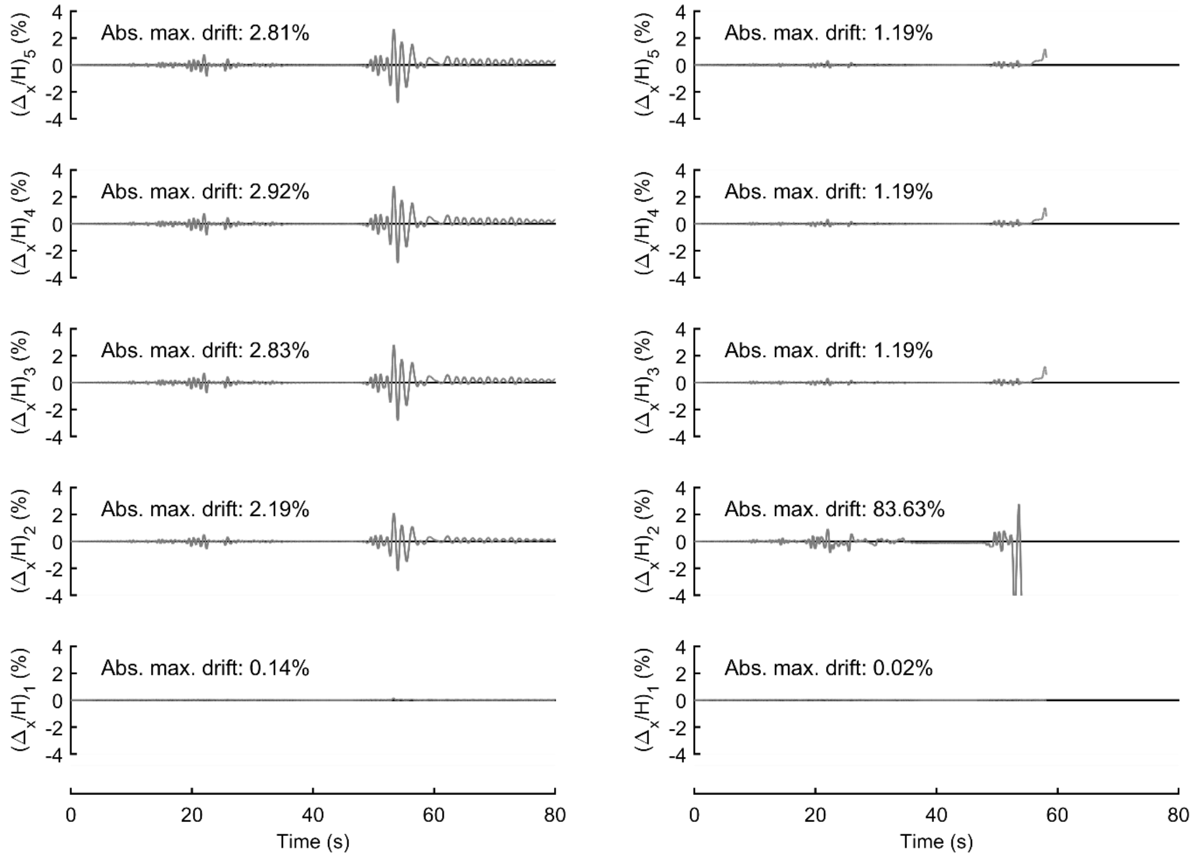
3.2 Nonlinear Response-History Analysis

Two, three-dimensional models of the Pyne Gould Corporation Building were constructed in *OpenSees* [5] to evaluate the nonlinear response-history analysis procedure for shear-wall buildings. The models were nominally identical, except one model had linear-elastic wall response in shear, while the other had nonlinear wall response in shear; only the latter is compliant with ASCE/SEI 41-17. The beams were modeled as fiber sections and the columns were modeled using the fiber-limit state approach. These columns were only designed to support gravity loads; however, this approach is still valid for secondary lateral-force-resisting members. The diaphragm was modeled as rigid. Fig. 9 shows that the first mode shape of the model is consistent with the observed collapse mechanism. Each building model was analyzed with the 2010 Darfield and 2011 Christchurch Earthquakes in sequence as recorded at the REHS station recordings.

The story-drift response histories for the earthquake sequence are shown in Fig. 11, where two different collapse mechanisms can be observed. When the wall response is linear-elastic in shear, flexural hinging and axial failure of the wall occurs, resulting in the strongback mechanism that enforces nearly equal drift demand along the height (see Fig. 10a). While these demands are not extreme (2-3% drift), they are large enough to induce axial failure in the columns based on the prescribed modeling parameters. When the wall response is nonlinear, as required by nonlinear analysis procedures in ASCE/SEI 41-17, the wall fails in shear at the second story and collapses in a soft-story mechanism (see excessively large maximum drift in Fig. 10b). While this is clearly a severe and undesirable response, there is no obvious indication that the upper stories would sustain damage, because the correct failure mode is not predicted.

4. Conclusions

The demand for seismic retrofit is growing in order to ensure the safety and, potentially, functionality of existing RC building infrastructure. ASCE/SEI 41-17 has been established as a standard code and practical tool for engineers to evaluate retrofit need and strategy for these buildings. The Tier 3 evaluation procedures provide substantial guidance for model development and component acceptance criteria to enable this work. As part of the broader ATC-134 project, two nonductile RC buildings that had well-documented earthquake-induced damage were modeled to assess the predictive capabilities of models developed under the ASCE/SEI 41-17 procedure. The Nanhua District Office was evaluated using linear and nonlinear response-history analysis procedures; the latter was performed using traditional moment-rotation springs and newly developed fiber-limit state hinges that evolve the ASCE/SEI 41-17 backbone curve throughout the analysis. The Pyne Gould Corporation Building was then evaluated using the nonlinear response-history analysis procedure with and without nonlinearity in the wall shear response. In all cases, the best estimate for the event ground motion(s) was used. The following conclusions can be drawn from these results:



(a) Elastic wall response in shear (b) Nonlinear wall response in shear

Fig. 10 – Pyne Gould Corporation Building story-drift response histories

- The linear response-history analysis procedure can identify vulnerable components well but not necessarily the nature of the vulnerability. In such cases, nonlinear analysis is required.
- A fiber-limit state model can be employed to produce similar magnitude of results to a moment-rotation spring model. The former requires no iteration whereas the latter requires a potentially time-consuming series of analyses to converge on an axial load ratio.
- The prescribed backbone curves underpredict damage observed in the Nanhua District Office regardless of modeling approach. Recalibration for short columns may be required.
- The collapse mode of the Pyne Gould Corporation Building is well predicted when shear failure of the wall is neglected, but this is not permitted by ASCE/SEI 41-17. This could indicate that the shear strength of flanged walls should be re-examined.

Several limitations to these conclusions must be recognized. Complete material-property data were not available for each building, and several known construction issues (e.g., presence of drainage pipes in columns or misplaced reinforcing steel) were not considered. Further, there is considerable uncertainty with respect to the actual site ground motions. Nonetheless, these buildings are sufficiently regular in geometry and configuration such that reasonably accurate response predictions using the ASCE/SEI 41-17 modeling procedures should be achievable. To help understand this shortcoming, future work is planned to investigate the effects of modeling-parameter uncertainty on response prediction.

5. Acknowledgments

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